

Influence of Joint Panel Zone on Seismic Behaviour of Beam-to-Column Connections

Arnav Anuj Kasar, Rupen Goswami, S.D. Bharti and M.K. Shrimali

Abstract Steel structures, because of their inherent ductility, are considered to be suitable for construction in areas of high seismicity. To ensure a certain expected performance of a structure, concept of strong column weak beam design was introduced and became widely accepted. Effect of strength of joint panel zone (JPZ) on the overall behaviour of strong column weak beam interior beam-to-column strong axis connection is studied, to understand their contribution on the performance of such connections. Three dimensional non-linear finite element analysis of three typical interior beam column joint subassemblages is carried out to study inelastic behaviour of such joints with different column to beam strength ratio. The results indicates that there is significant JPZ deformation leading to the kinking of columns before plastic hinges are formed in the beams even when the beams are weaker than the columns. This is due to inadequate shear strength of the JPZ. Also, excessive kinking in columns may lead to brittle failure of connections far before flexural capacity of beam is reached, thereby causing complete collapse of the structure.

Keywords Panel zone shear strength · Beam-to-column connections · Deformation · Plastic hinges · Kinking of column

1 Introduction

In India, demands of steel construction (i.e., steel structures) have increased significantly in recent years. This is because steel, as a material, possesses high strength-to-weight ratio and material ductility, and is a common choice in countries

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like USA and Japan as construction material. Thus, steel structures are considered to be relatively more ductile than those constructed using any other material. However, performance of steel structures during Northridge Earthquake, 1994 and Kobe Earthquake, 1995 forced researchers to reinvestigate seismic design of steel structures for improved behaviour. The most popular connection scheme employed for beam-to-column strong-axis connection was bolted web welded flange (BWFF) connection. After 1994 Northridge earthquake, it was observed that, one of the key issues was failure in the form of brittle fractures in beam-to-column connection welds [14], which can be attributed to large inelastic strain demand on complete joint penetration (CJP) welds connecting beam flanges to column flanges. Some factors that cause large inelastic strain demand on CJP welds are unreinforced connections, defects in welds and welding procedure (size and shape of weld access holes, backing bar etc.), kinking of column flanges, and yielding of joint panel zone (JPZ). Of these, “controlled inelastic deformation” of JPZ was originally considered to be good structural response.

A number of experimental and analytical investigations have been carried out, since late 1960s, to understand the behaviour of JPZ [10, 13]. These studies suggest that, when subjected to repeated cyclic distortions, yielding of JPZ is a stable phenomenon, and can be helpful in dissipating the energy induced by these distortions. But, it is also evident from these studies that the overall frame stiffness is greatly influenced by the stiffness of the JPZ. Thus, a balanced JPZ design is adopted, which aims at simultaneous onset of flexural hinging in beams and shear yielding in JPZ [7]. Further, in cases where the column is not strong enough in comparison to the beam, weak JPZ, despite of having stable post-yield response, causes large inelastic drifts and loss of overall stiffness of frames, and often leads to kinking of column flanges at the level of beam flanges. This increases curvature and causes failure of welded connections. Indian specifications for design of steel structures needs to put emphasis on this important aspect related to design and detailing of JPZ in steel structures.

2 Background

Steel beam to column joint in moment frames [6] are generally assumed to be rigid when overall analysis or design is carried out of the structure [16]. But the actual behaviour of these joints can be most closely predicted by assuming them to behave in a semi-rigid manner. A critical part of the joint is the JPZ. The overall behaviour of the JPZ along with connections are often classified as rigid, semi-rigid or flexible [4, 9].

Classical seismic design of moment frames depend on inelastic energy dissipation by component elements. These components are the beam, the beam-to-column connections, the JPZ, the column outside the joint, or a combination of these [11].

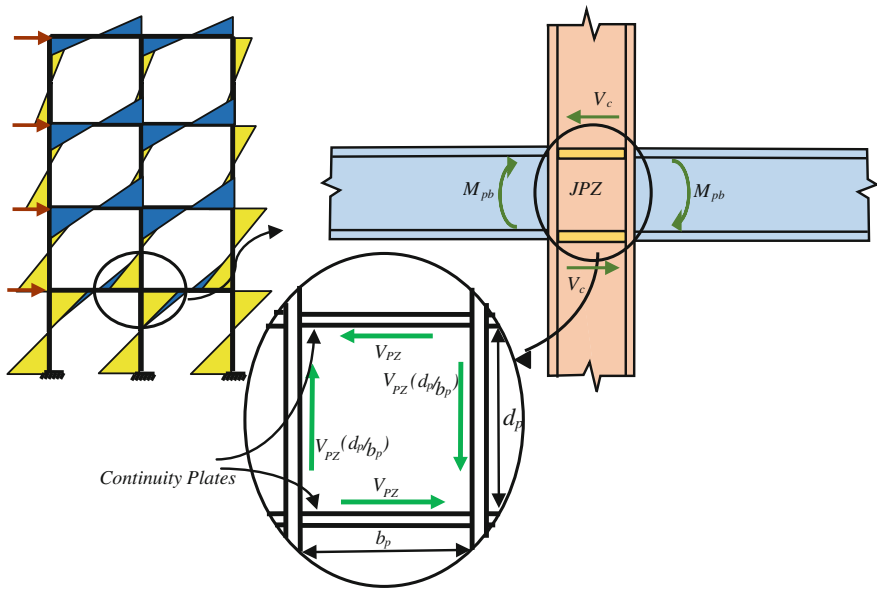


Fig. 1 Forces generated in JPZ in moment frame under lateral seismic actions

Concentration of inelastic action in the column outside the joint is not desired as it can prove to be detrimental to the overall stability of the structure. Similarly, concentration of inelastic action in the connections is detrimental may lead to brittle fractures as observed in past earthquakes. But, controlled ductile shear yielding of the JPZ may be allowed, and sometimes is beneficial to seismic response of the moment frame. However, too flexible JPZ causes secondary effects like kinking of column flange and high interstory drift; the later can cause severe non-structural damage [13]. Based on these, there are three distinct possible philosophies for design of the JPZs. These are, (i) *Strong Panel Zone Design* wherein the JPZ is supposed to remain elastic, forcing inelasticity in the beams; (ii) *Weak Panel Zone Design* wherein inelasticity is confined to the JPZ; and (iii) *Balanced Panel Zone Design* wherein the JPZ is designed such that inelasticity in the JPZ follows inelasticity in the beam [3, 5].

A typical interior beam column joint of a moment resisting frame along with the forces in JPZs is shown in Fig. 1; a JPZ is mostly subjected to pure shear, and produces stable hysteretic shear force-shear deformation response curves under reversed cyclic loading, as during seismic shaking. Controlled yielding of JPZ provides large energy dissipation capacity to the frame, but also leads to large overall deformation [13, 15]. Thus, weak panel zone design, though sometimes used in low-rise buildings, is not suitable for tall buildings.

3 Design of JPZ

The design shear strength (capacity) of JPZ is given by [1]

$$V_d = \phi_v 0.6 F_y d_c t_w, \text{ for } 0 < P \leq P_c \tag{1}$$

$$V_d = \phi_v 0.6 F_y d_c t_w \left(1.4 - \frac{P}{P_c} \right), \text{ for } 0.4 P_c < P \leq P_c \tag{2}$$

When frame stability including JPZ deformation is not considered. When frame stability including JPZ deformation is considered, the design strength is given by

$$V_d = \phi_v 0.6 F_y d_c t_w \left(1 + \frac{3 b_{cf} t_{cf}^2}{d_b d_c t_w} \right), \text{ for } 0; P \leq 0.75 \leq P_c, \text{ or} \tag{3}$$

$$V_d = \phi_v 0.6 F_y d_c t_w \left(1 + \frac{3 b_{cf} t_{cf}^2}{d_b d_c t_w} \right) \left(1.9 - \frac{1.2 P}{P_c} \right), \text{ for } 0.75 P_c < P \leq P_c, \tag{4}$$

Here, V_d is design shear capacity of JPZ, ϕ_v is resistance factor (in LRFD), F_y is minimum specified yield strength of column material, d_c is depth of column section, t_w is thickness of web of column section, P is factored axial load on the column, P_c is $0.6 P_y$ where P_y is the squash load of column, b_{cf} and t_{cf} are width and thickness of the column flange respectively, and d_b is the depth of deeper beam framing into the joint.

The shear force on a JPZ in an interior beam column joint of a moment frame, as shown in Fig. 1, is given by

$$V_{pz} = \frac{\sum M_{pb}}{(d_b - t_{bf})} \left(1 - \frac{(d_b - t_{bf})}{(H - d_b)} \right), \tag{5}$$

where, H is the height of column between two stories, $\sum M_{pb}$ is sum of resisting moments of beams framing in to the joint, and P_y is the squash load of the column. Thus, the minimum thickness t_{pz} of JPZ (including the thickness of doubler plates, if any) that is required to prevent yielding of JPZ is given by

$$t_{pz} \geq \frac{1}{F_y \left\{ 1 - \left(\frac{P}{P_c} \right)^2 \right\}^{1/2}} \frac{\sum M_{pb}}{(d_c - 2 t_{cf})(d_b - t_{bf})} \left(1 - \frac{(d_b - t_{bf})}{(H - d_b)} \right). \tag{6}$$

This assumes uniform distribution of shear stress within JPZ that is proved to be valid by many experimental investigations [2, 10, 11]. Also, this is the minimum thickness that will ensure that shear yielding in JPZ initiates only after flexural yielding of beams [12].

But, often design codes and guidelines directly prescribe minimum thickness of JPZ to be provided, instead of guidelines to design the JPZ. For instance, the required minimum thickness of JPZ for simultaneous shearyielding of panel zone and flexural yielding of beam is as

$$t = \frac{C_y M_c \left(\frac{H-d_b}{H} \right)}{(0.9)0.6F_y R_{yC} d_c (d_b - t_{bf})}, \quad (7)$$

where, R_{yC} is ratio of expected yield strength of column material to minimum specified yield strength, and M_c and C_y are coefficients to account for moment at column centreline resulting from moment amplification due to plastic hinging in beam and strain hardening of material of column, respectively. Alternatively, the required minimum thickness of the JPZ is given by

$$t \geq (d_p + b_p)/90, \quad (8)$$

where, d_p is panel zone depth between continuity plates, and b_p is the width of the panel zone between column flanges.

4 Structural Models and Analyses

Three different combinations of strong column weak beam (SCWB) strong-axis joint subassemblages are selected and analysed to illustrate JPZ behaviour; column to beam strength ratio $\sum M_{pc}/\sum M_{pb}$ considered are 1.1, 1.2 and 2.1. Each beam-column combination is used to model an interior beam column joint, as per specifications given in Indian Standard 800 [9], with continuity plates to reinforce the joints. The beams, columns and continuity plates are assumed to be of grade ASTM A36 steel with isotropic hardening model (yield stress of 250 MPa and ultimate stress of 415 MPa). The height of columns considered in the subassemblages is 3.8 m, which, in most cases, is the average storey height (Fig. 2). The distance considered between column centreline and the point of application of load on beams is 3.0 m, representing span of beam. AISC wide flange plastic sections are used for beams and columns. The thickness of continuity plates are considered to be equal to beam flange thickness, length equal to the clear distance between column flanges, and width equal to clear flange width of the column.

Nonlinear analyses are carried out on 3-dimensional solid models of the interior joint subassemblages using finite element analysis software ABAQUS [8]. A uniform mesh is developed for the models using eight noded linear brick element (C3D8R). SAC's standard loading protocol (Fig. 3) is used for analyses to obtain the differences in responses of these beam column joints. Axial compressive load is not considered on the columns, to reduce the number of parameters on which shear capacity of JPZ depends.

Fig. 2 Typical interior beam-column joint subassemblage

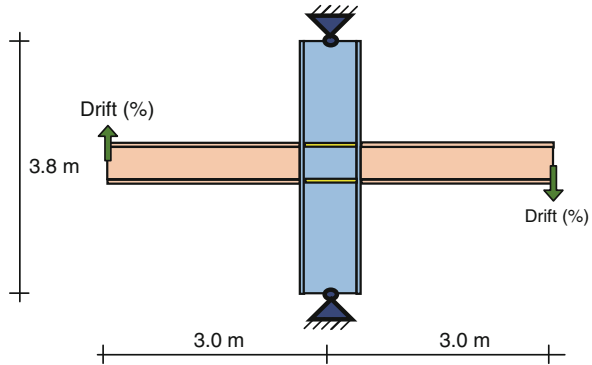
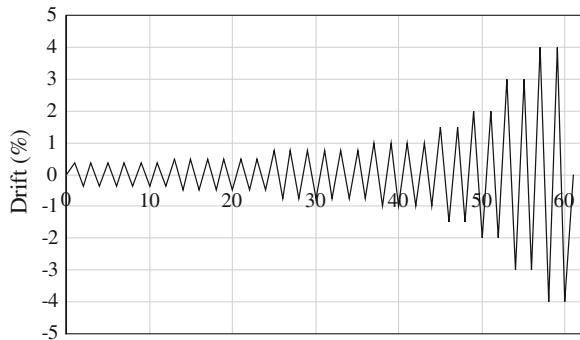


Fig. 3 Multi-cycle standard loading protocol



5 Results and Discussion

Results of nonlinear finite element analyses indicate that significant shear yielding occurs in panel zones in all three beam-column joint subassemblages without formation of flexural plastic hinge in beams. Yielding of JPZ occurs despite satisfying two key requirements of design codes, namely SCWB design and minimum thickness of JPZ, as listed in Table 1. This indicates that explicit design provisions are required to ensure plastic hinge formation in beam prior to shear yielding of JPZ. First yield in JPZ occurs at beam end drift of about 0.75 % (drift of 22.5 mm; rotation of 0.0075 rad).

Figure 4 shows the state of shear stress (in MPa) at 4 % drift level in the joint of the three subassemblages; complete yielding of JPZs in shear is seen (yield strength in shear being 145 MPa). Also, there is concentration of high (negative) shear at the beam flange levels, which are usually conceived to carry only flexure. Further, kinking of columns is evident due to shear distortion of the JPZ in all the three subassemblages, irrespective of the column to beam strength ratio.

Figure 5 shows von Mises stress (in MPa) at 4 % drift level in the joint of the three subassemblages; yielding of beam flanges is seen due to large normal stress.

Table 1 Performance of selected interior joint subassemblages

#	Column	Beam	$\frac{M_{FC}}{M_{FB}}$	$L_{JPZ, reqd}$ (mm)	$L_{JPZ, prov}$ (mm)	$V_{CAPACITY}$ (kN)	V_{DEMAND} (kN)	$\frac{V_{DEMAND}}{V_{CAPACITY}}$	Yielding of JPZ
1	W14×257	W18×192	1.1	9	48	1,877	6,537	3.5	Yes
2	W18×86	W24×62	1.2	11	20	841	1,744	2.1	Yes
3	W24×131	W21×73	2.1	12	24	1,400	2,281	1.6	Yes

But, the level of inelasticity in the beam is nominal in the subassemblages with column to beam strength ratios of 1.1. Yielding of flanges of the beams progressively increases with increase in column to beam strength ratio. But, the local yielding of columns is also seen, with least yielding of column in the subassemblage with column to beam strength ratio of 2.1. This indicates that higher column to beam strength ratio is required as yielding of column is not acceptable.

Further, it is also observed that large inelastic deformation JPZs causes large inelastic strain demand at weld regions in all the three subassemblages, irrespective of the value of column to beam strength ratio. This leads to brittle fracture of CJP welds connecting the beam flanges to the columns, as was observed in the Northridge and Kobe earthquakes. It is also pertinent to mention that premature yielding of JPZs in shear leading to kinking of column flanges at the beam flange levels restrains the beams in the joint region from developing their full flexural plastic capacity, thereby nullifying the whole SCWB design intent, although large amount of energy dissipation is possible due to inelastic shear deformation of JPZs.

The maximum shear demand to shear capacity of the three JPZs is in the range 1.6–3.5 at drift level of 4 %, as shown in Table 1. These results indicate that the shear strengths of JPZs are inadequate for all values of column to beam strength ratio. Thus, the minimum thickness of JPZ requirement given in IS 800 is not sufficient to prevent yielding of JPZs prior to formation of flexural plastic hinge in beams—JPZs need to be designed for the maximum expected demand as given by Eq. 5. Unless the JPZs are designed not to yield prior to yielding of beams, rigid connection behaviour is not realisable as envisaged by IS 800, particularly in special moment frames. Also, the criterion to ensure SCWB design alone does not ensure that plastic hinge will form in the beam before yielding of the JPZs.

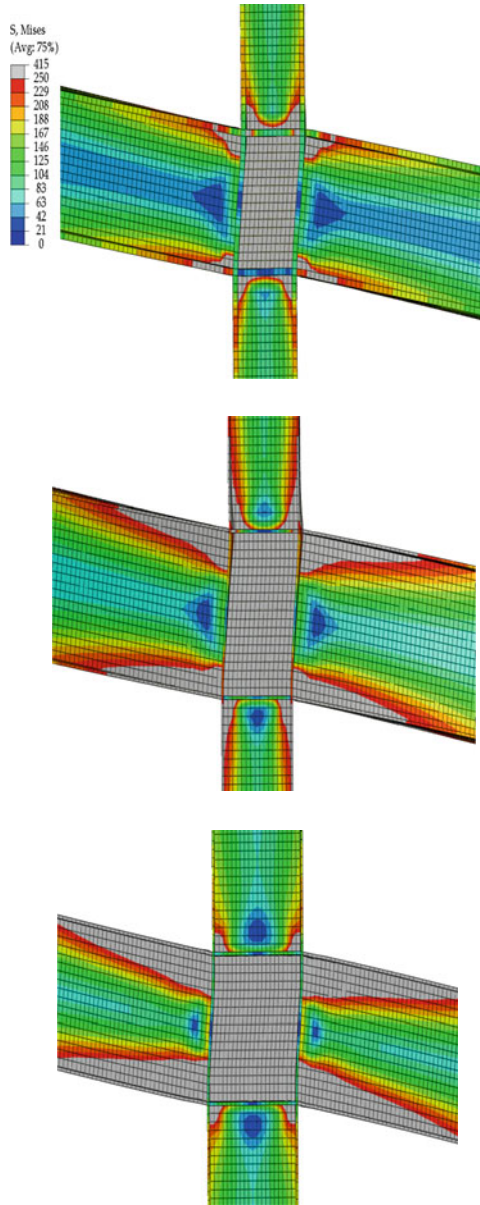
Figure 6 shows the variation of shear demand on JPZ normalized with shear capacity of JPZ attained as per SAC’s standard loading protocol; the hysteresis response curves corresponds to different values of column to beam strength ratio. The response curves show stable inelastic load deformation response in all three joint subassemblages, irrespective of the column to beam strength ratio. This is representative of stable yielding behaviour of JPZs in shear.



W18×192 Beam and W14×257 Column
W24×62 Beam and W18×86 Column
W21×73 Beam and W24×131 Column

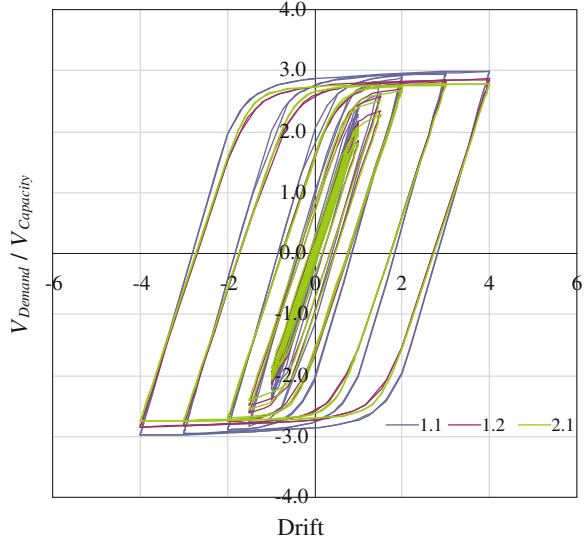
Fig. 4 Distribution of shear stress (in MPa) at 4 % drift

Fig. 5 Distribution of von Mises stress (in MPa) at 4 % drift



W18x192 Beam and W14x257 Column
W21x73 Beam x W24x131 Column
W24x62 Beam and W18x86 Column

Fig. 6 Hysteresis curves showing variation of normalised shear in JPZs under multi-cycle standard loading protocol



6 Conclusion

Premature yielding of JPZs limits formation of plastic flexural hinges in beams, even when SCWB design philosophy is employed. This indicates that JPZs need to be strengthened to resist the shear demand produced. Thus, it is concluded that the strength hierarchy between columns and beams is a necessary but not sufficient condition to ensure flexural yielding of beams. Design provisions in IS 800 need to include strength design of JPZs to avoid premature yielding of JPZs.

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